10.24.3.5 Where AASHTO M 164 (ASTM A 325) bolts of any diameter or AASHTO M 253 (ASTM A 490) bolts equal to or less than 1 inch in diameter are to be installed in an oversize or short slotted hole in an outer ply, a hardened washer conforming to ASTM F 436 shall be used.

10.24.3.6 When AASHTO M 253 (ASTM A 490) bolts over 1 inch in diameter are to be installed in an oversize or short slotted hole in an outer ply, hardened washers conforming to ASTM F 436 except with $^5/_{16}$ inch minimum thickness shall be used under both the head and the nut in lieu of standard thickness hardened washers. Multiple hardened washers with combined thickness equal to or greater than $^5/_{16}$ inch do not satisfy this requirement.

10.24.3.7 Where AASHTO M 164 (ASTM A 325) bolts of any diameter or AASHTO M 253 (ASTM A 490) bolts equal to or less than 1 inch in diameter are to be installed in a long slotted hole in an outer ply, a plate washer or continuous bar of at least $^5/_{16}$ inch thickness with standard holes shall be provided. These washers or bars shall have a size sufficient to completely cover the slot after installation and shall be of structural grade material but need not be hardened except as follows. When AASHTO M 253 (ASTM A 490) bolts over 1 inch in diameter are to be used in long slotted holes in external plies, a single hardened washer conforming to ASTM F436 but with $^5/_{16}$ inch minimum thickness shall be used in lieu of washers or bars of structural grade material. Multiple hardened washers with combined thickness equal to or greater than $^5/_{16}$ inch do not satisfy this requirement.

10.24.4 Size of Fasteners (Rivets or High-Strength Bolts)

10.24.4.1 Fasteners shall be of the size shown on the drawings, but generally shall be $^3/_4$ inch or $^7/_8$ inch in diameter. Fasteners $^5/_8$ inch in diameter shall not be used
+ in members carrying design loads except in $2^1/_2$-inch legs of angles and in flanges of sections.

10.24.4.2 The diameter of fasteners in angles car-
+ rying design loads shall not exceed one-fourth the width of the leg in which they are placed.

10.24.4.3 In angles whose size is not determined
+ by design loads, $^5/_8$-inch fasteners may be used in 2-inch legs, $^3/_4$-inch fasteners in $2^1/_2$-inch legs, $^7/_8$-inch fasteners in 3-inch legs, and 1-inch fasteners in $3^1/_2$-inch legs.

10.24.4.4 Structural shapes which do not admit the use of $^5/_8$-inch diameter fasteners shall not be used except in handrails.

10.24.5 Spacing of Fasteners

10.24.5.1 Pitch and Gage of Fasteners

The pitch of fasteners is the distance along the line of principal stress, between centers of adjacent fasteners, +
measured along one or more fastener lines. The gage of fasteners is the distance between adjacent lines of fasten- +
ers or the distance from the back of angle or other shape to the first line of fasteners.

10.24.5.2 Minimum Spacing of Fasteners

The minimum distance between centers of fasteners in standard holes shall be three times the diameter of the fastener but, preferably, shall not be less than the following:

Fastener Diameter (in.)	Minimum Spacing (in.)	
		+
1	$3^1/_2$	+
$^7/_8$	3	+
$^3/_4$	$2^1/_2$	+
$^5/_8$	$2^1/_4$	+

10.24.5.3 Minimum Clear Distance between Holes

When oversize or slotted holes are used, the minimum clear distance between the edges of adjacent holes in the direction of the force and transverse to the direction of the force shall not be less than twice the diameter of the bolt.

10.24.5.4 Maximum Spacing of Fasteners

The maximum spacing of fasteners shall be in accordance with the provisions of Articles 10.24.6, as applicable.

10.24.6 Maximum Spacing of Sealing and Stitch Fasteners

10.24.6.1 Sealing Fasteners

For sealing against the penetration of moisture in joints, the fastener spacing along a single line of fasteners adjacent to a free edge of an outside plate or shape shall not exceed 4 inches + $4t$ or 7 inches. If there is a second line of fasteners uniformly staggered with those in the line adjacent to the free edge, at a gage "g" less than $1^1/_2$ inches + $4t$ therefrom, the staggered pitch in two such lines, considered together, shall not exceed 4 inches + $4t$ – $3g/4$ or 7 inches, but need not be less than one-half the requirement for a single line, where t is the thickness of the thinner outside plate or shape (in.), and g is the gage between fasteners (in.).

10.24.6.2 Stitch Fasteners

In built-up members where two or more plates or shapes are in contact, stitch fasteners shall be used to ensure that the parts act as a unit and, in compression members, to prevent buckling. In compression members the pitch of stitch fasteners on any single line in the direction of stress shall not exceed $12t$, except that, if the fasteners on adjacent lines are staggered and the gage, g, between the line under consideration and the farther adjacent line (if there are more than two lines) is less than $24t$, the staggered pitch in the two lines, considered together, shall not exceed $12t$ or $15t – 3g/8$. The gage between adjacent lines of fasteners shall not exceed $24t$. In tension members the pitch shall not exceed twice that specified for compression members and the gage shall not exceed that specified for compression members.

The maximum pitch of fasteners in built-up members shall be governed by the requirements for sealing or stitch fasteners, which is minimum.

For pitch of fasteners in the ends of compression members, see Article 10.16.13.

10.24.7 Edge Distance of Fasteners

10.24.7.1 General

The distance from the center of any fastener in a standard hole to an edge of a connected part shall not be less than the applicable value specified in Table 10.24.7.1.A.

The maximum distance from the center of any fastener to any edge shall be eight times the thickness of the thinnest outside plate, but shall not exceed 5 inches.

10.24.7.2 When there is only a single transverse fastener in the direction of the line of force in a standard or short slotted hole, the distance from the center of the hole to the edge of the connected part shall not be less than $1^1/_2$ times the diameter of the fastener, unless accounted for by the bearing provisions of Table 10.32.3B or Article 10.56.1.3.2.

10.24.7.3 When oversize or slotted holes are used, the distance between edges of holes and edges of members shall not be less than the diameter of the bolt.

TABLE 10.24.7.1A Minimum Edge Distance from Center of Standard Hole to Edge of Connected Part

Fasteners Diameter (in.)	At Sheared or Thermally Cut Edges (in.)	At Rolled or Planed Edges (in.)	At Flange Edges of Beams and Channels (in.)
1	$1^3/_4$	$1^1/_2$	$1^1/_4$
$^7/_8$	$1^1/_2$	$1^1/_4$	$1^1/_8$
$^3/_4$	$1^1/_4$	$1^1/_8$	1
$^5/_8$	$1^1/_8$	1	$^7/_8$

10.24.8 Long Rivets

Rivets subjected to design forces and having a grip in excess of $4^1/_2$ diameters shall be increased in number at least 1 percent for each additional $^1/_{16}$ inch of grip. If the grip exceeds six times the diameter of the rivet, specially designed rivets shall be used.

10.25 LINKS AND HANGERS

10.25.1 Net Section

In pin-connected tension members other than eyebars, the net section across the pin hole shall be not less than 140 percent, and the net section back of the pin hole not less than 100 percent of the required net section of the body of the member. The ratio of the net width (through the pin hole transverse to the axis of the member) to the thickness of the segment shall not be more than 8. Flanges not bearing on the pin shall not be considered in the net section across the pin.

Pin plates are not recommended in new construction. The thickness required shall be full length. Hanger plates shall be designed to provide free movement of the parts.

10.25.2 Location of Pins

Pins shall be so located with respect to the gravity axis of the members as to reduce to a minimum the stresses due to bending.

10.25.3 Size of Pins

Pins shall be proportioned for the maximum shears and bending moments produced by the members connected. If there are eyebars among the parts connected, the diameter of the pin shall be not less than

$$\left[\frac{3}{4} + \frac{F_y}{400{,}000}\right] b_{eb} \qquad (10\text{-}11)$$

where:

F_y = specified minimum yield strength of steel (psi)
b_{eb} = width of the body of the eyebar (in.)

10.25.4 Pin Plates

When necessary for the required section or bearing area, the section at the pin holes shall be increased on each segment by plates so arranged as to reduce to a minimum the eccentricity of the segment. One plate on each side shall be as wide as the outstanding flanges will allow. At least one full-width plate on each segment shall extend to the far edge of the stay plate and the others not less than 6 inches beyond the near edge. These plates shall be connected by enough rivets, bolts, or fillet welds to transmit the bearing pressure, and so arranged as to distribute it uniformly over the full section.

10.25.5 Pins and Pin Nuts

10.25.5.1 Pins shall be of sufficient length to secure a full bearing of all parts connected upon the turned body of the pin. They shall be secured in position by hexagonal recessed nuts or by hexagonal solid nuts with washers. If the pins are bored, through rods with cap washers may be used. Pin nuts shall be malleable castings or steel. They shall be secured by cotter pins or other suitable locking devices which will not affect the removal of the nut.

10.25.5.2 Members shall be restrained against lateral movement on the pins and against lateral distortion due to the skew of the bridge.

10.26 UPSET ENDS

Bars and rods with screw ends, where specified, shall be upset to provide a section at the root of the thread, which will exceed the net section of the body of the member by at least 15 percent.

10.27 EYEBARS

10.27.1 Thickness and Net Section

Eyebars shall be of a uniform thickness without reinforcement at the pin holes. The thickness of eyebars shall be not less that $^1/_8$ of the width, nor less than $^1/_2$ inch, and not greater than 2 inches. The section of the head through the center of the pin hole shall exceed the required section of the body of the bar by at least 35 percent. The net section back of the pin hole shall not be less than 75 percent of the required net section of the body of the member. The radius of transition between the head and body of the eyebar shall be equal to or greater than the width of the head through the centerline of the pin hole.

10.27.2 Packing of Eyebars

10.27.2.1 The eyebars of a set shall be symmetrical about the central plane of the truss and as nearly parallel as practicable. Bars shall be as close together as practicable and held against lateral movement, but they shall be so arranged that adjacent bars in the same panel will be separated by at least $^1/_2$ inch.

10.27.2.2 Intersecting diagonal bars not far enough apart to clear each other at all times shall be clamped together at the intersection.

10.27.2.3 Steel filling rings shall be provided, if needed, to prevent lateral movement of eyebars or other members connected on the pin.

10.28 FORKED ENDS

Forked ends will be permitted only where unavoidable. There shall be enough pin plates on forked ends to make the section of each jaw equal to that of the member. The pin plates shall be long enough to develop the pin plate beyond the near edge of the stay plate, but not less than the length required by Article 10.25.4.

10.29 FIXED AND EXPANSION BEARINGS

10.29.1 General

10.29.1.1 Fixed ends shall be firmly anchored. Bearings for spans less than 50 feet need have no provision for rotation. Spans of 50 feet or greater shall be provided with a type of bearing to accommodate rotation.

10.29.1.2 Expansion ends shall be provided with a type of bearing to accommodate rotation and expansion.

10.29.1.3 *Deleted*

10.29.2 *Deleted*

10.29.3 *Deleted*

10.29.4 Sole Plates and Masonry Plates

10.29.4.1 Sole plates and masonry plates shall have a minimum thickness of $^{3}/_{4}$ inch.

10.29.4.2 For spans on inclined grades greater than 1 percent without hinged bearings, the sole plates shall be beveled so that the bottom of the sole plate is level, unless the bottom of the sole plate is radially curved.

10.29.5 Masonry Bearings

Beams, girders, or trusses on masonry shall be so supported that the bottom chords or flanges will be above the bridge seat, preferably not less than 6 inches.

10.29.6 Anchor Rods

10.29.6.1 Trusses, girders, and rolled beam spans preferably shall be securely anchored to the substructure. Anchor rods shall be headed, hooked, or threaded with a nut to secure a satisfactory grip upon the material used to embed them in the holes. All anchor rods shall conform to specifications shown in Table 10.2C. High strength steels (quenched and tempered) are not recommended for use in hooked anchor rods since bending with heat may affect their strength. The embedded end of a threaded rod with a nut shall have a positive locking device or system to prevent rod rotation when a nut is installed on other end.

10.29.6.2 The following are the minimum requirements for each bearing:

For rolled beam spans the outer beams shall be anchored at each end with 2 rods, 1 inch in diameter, set 10 inches in the masonry.

For trusses and girders:

Spans 50 feet in length or less; 2 rods, 1 inch in diameter set 10 inches in the masonry.

Spans 51 to 100 feet; 2 rods, $1^{1}/_{4}$ inches in diameter, set 12 inches in the masonry.

Spans 101 to 150 feet; 2 rods, $1^{1}/_{2}$ inches in diameter, set 15 inches in the masonry.

Spans greater than 150 feet; 4 rods, $1^{1}/_{2}$ inches in diameter, set 15 inches in the masonry.

10.29.6.3 Anchor rods shall be designed to resist uplift as specified in Article 3.17 and seismic forces specified in Article 3.21. Other restraining devices may be used in conjunction with anchor rods.

10.29.7 Pedestals and Shoes

10.29.7.1 Pedestals and shoes preferably shall be made of cast steel or structural steel. The difference in width between the top and bottom bearing surfaces shall not exceed twice the distance between them. For hinged bearings, this distance shall be measured from the center of the pin. In built-up pedestals and shoes, the web plates and angles connecting them to the base plate shall be not less than $^{5}/_{8}$ inch thick. If the size of the pedestal permits, the webs shall be rigidly connected transversely. The minimum thickness of the metal in cast steel pedestals shall be 1 inch. Pedestals and shoes shall be so designed that the load will be distributed uniformly over the entire bearing.

10.29.7.2 Webs and pin holes in the webs shall be arranged to keep any eccentricity to a minimum. The net section through the hole shall provide 140 percent of the net section required for the design load transmitted through the pedestal or shoe. Pins shall be of sufficient length to secure a full bearing. Pins shall be secured in position by appropriate nuts with washers. All portions of pedestals and shoes shall be held against lateral movement of the pins.

10.30 FLOOR SYSTEM

10.30.1 Stringers

Stringers preferably shall be framed into floor beams. Stringers supported on the top flanges of floor beams preferably shall be continuous.

10.30.2 Floor Beams

Floor beams preferably shall be at right angles to the trusses or main girders and shall be rigidly connected thereto. Floor beam connections preferably shall be located so the lateral bracing system will engage both the floor beam and the main supporting member. In pin-connected trusses, if the floor beams are located below the bottom chord pins, the vertical posts shall be extended sufficiently below the pins to make a rigid connection to the floor beam.

10.30.3 Cross Frames

In bridges with wooden floors and steel stringers, intermediate cross frames (or diaphragms) shall be placed between stringers more than 20 feet long.

10.30.4 Expansion Joints

10.30.4.1 To provide for expansion and contraction movement, floor expansion joints shall be provided at all expansion ends of spans and at other points where they may be necessary.

10.30.4.2 Apron plates, when used, shall be designed to bridge the joint and to prevent, so far as practicable, the accumulation of roadway debris upon the bridge seats. Preferably, they shall be connected rigidly to the end floor beam.

10.30.5 End Floor Beams

There shall be end floor beams in all square-ended trusses and girder spans and preferably in skew spans. End floor beams for truss spans preferably shall be designed to permit the use of jacks for lifting the superstructure. For this case the allowable stresses may be increased 50 percent.

10.30.6 End Panel of Skewed Bridges

In skew bridges without end floor beams, the end panel stringers shall be secured in correct position by end struts connected to the stringers and to the main truss or girder. The end panel lateral bracing shall be attached to the main trusses or girders and also to the end struts. Adequate provisions shall be made for the expansion movement of stringers.

10.30.7 Sidewalk Brackets

Sidewalk brackets shall be connected in such a way that the bending stresses will be transferred directly to the floor beams.

10.30.8 Stay-in-Place Deck Forms

10.30.8.1 Concrete Deck Panels

When precast prestressed deck panels are used as permanent forms spanning between beams, stringers, or girders, the requirements of Article 9.12, Deck Panels, and Article 9.23, Deck Panels, shall be met.

10.30.8.2 Metal Stay-in-Place Forms

When metal stay-in-place forms are used as permanent forms spanning between beams, stringers, or girders, the forms shall be designed a minimum of, to support the weight of the concrete (including that in the corrugations, if applicable), a construction load of 50 psf, and the weight of the form. The forms shall be designed to be elastic under construction loads. The elastic deformation caused by the dead load of the forms, plastic concrete and reinforcement, shall not exceed a deflection greater than $L/80$ or one half inch, for form work spans (L) of 10 feet or less, or a deflection of $L/240$ or three-quarter inch, for form work for spans L over 10 feet. Dead load due to metal stay-in-place forms shall be taken into account in design of girders.

Part C
Service Load Design Method

Allowable Stress Design

10.31 SCOPE

Allowable stress design is a method for proportioning structural members using design loads and forces, allowable stresses, and design limitations for the appropriate material under service conditions. See Part D – Strength Design Method – Load Factor Design for a preferred design procedure.

10.32 ALLOWABLE STRESSES

10.32.1 Steel

Allowable stresses for steel shall be as specified in Table 10.32.1A.

10.32.2 Weld Metal

Unless otherwise specified, the ultimate strength of weld metal shall be equal to or greater than specified minimum value of the base metal. Allowable stresses on the effective areas of weld metal shall be as follows:

Butt Welds

The same as the base metal joined, except in the case of joining metals of different yields when the lower yield material shall govern.

Fillet Welds

$$F_v = 0.27\,F_u \tag{10-12}$$

where:

F_v = allowable basic shear stress (psi)
F_u = tensile strength of the electrode classification (psi).

When detailing fillet welds for quenched and tempered steels the designer may use electrode classifications with strengths less than the base metal provided that this requirement is clearly specified on the plans.

10.32.3 Fasteners

Allowable stresses for fasteners shall be as listed in Tables 10.32.3A and 10.32.3B, and the allowable force on a slip-critical connection shall be as provided by Article 10.32.3.2.1.

10.32.3.1 General

10.32.3.1.1 In proportioning fasteners for shear or tension, the cross sectional area based upon the nominal diameter shall be used except as otherwise noted.

10.32.3.1.2 The effective bearing area of a fastener shall be its diameter multiplied by the thickness of the metal on which it bears. In metal less than $^3/_8$ inch thick, countersunk fasteners shall not be assumed to carry load. In metal $^3/_8$ inch thick and over, one-half of the depth of countersink shall be omitted in calculating the bearing area.

10.32.3.1.3 In determining whether the bolt threads are excluded from the shear planes of the contact surfaces, thread length of bolts shall be calculated as at least two thread pitches greater than the specified thread length as an allowance for thread run out.

10.32.3.1.4 In bearing-type connections, pull-out shear in a plate should be investigated between the end of the plate and the end row of fasteners. (See Table 10.32.3B, footnote g.).

10.32.3.1.5 Deleted

10.32.3.1.6 Joints, utilizing high-strength bolts, required to resist shear between their connected parts are designated as either slip-critical (see Article 10.24.1.4) or bearing-type connections. Shear connections subjected to stress reversal, or where slippage would be undesirable, shall be slip-critical connections. Potential slip of joints should be investigated at intermediate load stages especially those joints located in composite regions.

10.32.3.1.7 The percentage of stress increase shown in Article 3.22, Combination of Loads, shall apply to allowable stresses in bolted slip-critical connections using high-strength bolts, except that in no case shall the percentage of allowable stress exceed 133 percent, and the requirements of Article 10.32.3.3 shall not be exceeded.

10.32.3.1.8 Bolted bearing-type connections shall be limited to members in compression and secondary members.

10.32.3.2 The allowable stress in shear, bearing and tension for AASHTO M164 (ASTM A325) and AASHTO M253 (ASTM A490) bolts shall be as listed in Table 10.32.3B.

\+ High strength bolts installed according to the Standard
\+ Specifications of the California Department of Transpor-
\+ tation, Section 55, will be fully tensioned and the contact
\+ surface condition of the assembly will be Class B.

+ **TABLE 10.32.1A Allowable Stresses—Structural Steel (psi)**

+ AASHTO Designation		M 270 Grade 36	M 270 Grade 50	M 270 Grade 50W		M 270 Grades 100/100W	
Equivalent ASTM Designation		A 709 Grade 36	A 709 Grade 50	A 709 Grade 50W	A 709 Grade HPS 70W	A 709 Grades 100/100W	
Thickness of Plates		Up to 4" included	Up to 4" included	Up to 4" included	Up to 4" included	Up to $2^1/_2$" included	Over $2^1/_2$" to 4" included
+ Shapes		All Groups	All Groups	All Groups	N/A	N/A	
+ Axial tension in members with no holes for high-strength bolts or rivets.	0.55 F_y	20,000	27,000	27,000	38,000	N/A	
Use net section when member has any open holes larger than $1^1/_4$ inch diameter such as perforations.	0.46 F_u		N/A			51,000	46,000
+ Axial tension in members with holes for high-strength bolts or rivets and tension in extreme fiber of rolled	Gross Section[i] 0.55 F_y	20,000	27,000	27,000	38,000	N/A	
+ shapes girders, and built-up sections subject to bending. Satisfy both Gross and Net Section criterion.	Net Section 0.50 F_u	29,000	32,500	35,000	45,000	N/A	
	Net Section 0.46 F_u		N/A			51,000	46,000
Axial compression, gross section: stiffeners of plate girders. Compression in splice material, gross section		20,000	27,000	27,000	38,000	55,000	49,000
+ Compression in extreme fibers of doubly symmetrical I- and H-shape members with compact flanges continuously connected to the web and bent about their weak axes (except members with the yield strength greater than 65,000 psi); solid round and square bars; and solid rectangular sections bent about their weak axes	0.625 F_y	22,000	31,000	31,000	43,000	62,000	62,000
Compression in extreme fibers of rolled shapes, girders, and built-up sections subject to bending. Gross section, when compression flange is:							
(A) Supported laterally its full length by embedment in concrete	0.55 F_y	20,000	27,000	27,000	38,000	55,000	49,000
(B) Partially supported or is unsupported [a, b]							

$$F_b = \frac{50\times10^6 C_b}{S_{xc}}\left(\frac{I_{yc}}{l}\right)\sqrt{0.772\frac{J}{I_{yc}}+9.87\left(\frac{d}{l}\right)^2} \le 0.55F_y$$

TABLE 10.32.1A Allowable Stresses—Structural Steel (psi) (continued)

$$C_b = \frac{12.5M_{max}}{2.5M_{max} + 3M_A + 4M_B + 3M_c}$$

where:

M_{max} = absolute value of maximum moment in the unbraced beam segment (lb-in.)
M_A = absolute value of moment at quarter point of the unbraced beam segment (lb-in.)
M_B = absolute value of moment at midpoint of the unbraced beam segment (lb-in.)
M_C = absolute value of moment at three- quarter point of the unbraced segment (lb-in.)

C_b = 1.0 for unbraced cantilevers and for members where the moment within a significant portion of the unbraced segment is greater than or equal to the larger of the segment end moments.

Compression in concentrically loaded columns [c]							
with $C_c = (2\pi^2 E/F_y)^{1/2}$ =		126.1	107.0	107.0	90.4	75.7	79.8
when $KL/r \leq C_c$							
$F_a = \frac{F_y}{F.S.}\left[1 - \frac{(KL/r)^2 F_y}{4p^2 E}\right] =$		16,980 – 0.53 $(KL/r)^2$	23,580 – 1.03 $(KL/r)^2$	23,580 – 1.03 $(KL/r)^2$	33,020 – 2.02 $(KL/r)^2$	47,170 – 4.12 $(KL/r)^2$	42,450 – 3.33 $(KL/r)^2$
when $KL/r > C_c$							
$F_a = \frac{p^2 E}{F.S.(KL/r)^2} =$		$\frac{135{,}000{,}740}{(KL/r)^2}$					
with $F.S.$ = 2.12							
Shear in girder webs, gross section	$F_v = 0.33\ F_y$	12,000	17,000	17,000	23,000	33,000	30,000
Bearing on milled stiffeners and other steel parts in contact (rivets and bolts excluded)	$0.80\ F_y$	29,000	40,000	40,000	56,000	80,000	72,000
Stress in extreme fiber of pins [d]	$0.80\ F_y$	29,000	40,000	40,000	56,000	80,000	72,000
Shear in pins	$F_v = 0.40\ F_y$	14,000	20,000	20,000	28,000	40,000	36,000
Bearing on pins not subject to rotation [g]	$0.80\ F_y$	29,000	40,000	40,000	56,000	80,000	72,000
Bearing on pins subject to rotation (such as used in rockers and hinges)	$0.40\ F_y$	14,000	20,000	20,000	28,000	40,000	36,000
Bearing on connected material at Low Carbon Steel Bolts (ASTM A 307), Turned Bolts, Ribbed Bolts, and Rivets (ASTM A 502 Grades 1 and 2)—Governed by Table 10.32.3A							

+ **Footnotes for Table 10.32.1A Allowable Stresses—Structural Steel (psi)**

[a] For the use of larger C_b values, see Structural Stability Research Council *Guide to Stability Design Criteria for Metal Structures*, 3rd Ed., pg. 135. If cover plates are used, the allowable static stress at the point of theoretical cutoff shall be as determined by the formula.

[b] λ = length in inches, of unsupported flange between lateral connections, knee braces, or other points of support.

I_{yc} = moment of inertia of compression flange about the vertical axis in the plane of the web in.^{4}

d = depth of girder, in.

$J = \frac{\left[\left(bt^3\right)_c + \left(bt^3\right)_t + Dt_w^3\right]}{3}$ where b and t represent the flange width and thickness of the compression and tension flange, respectively (in.^{4}).

S_{xc} = section modulus with respect to compression flange (in.^{3}).

[c] E = modulus of elasticity of steel

r = governing radius of gyration

L = actual unbraced length

K = effective length factor (see Appendix C)

$F.S.$ = factor of safety = 2.12

For graphic representation of these formulas, see Appendix C.

The formulas do not apply to members with variable moment of inertia. Procedures for designing members with variable moments of inertia can be found in the following references: "Engineering Journal," American Institute of Steel Construction, January 1969, Volume 6, No. 1, and October 1972, Volume 9, No. 4; and "Steel Structures," by William McGuire, 1968, Prentice-Hall, Inc., Englewood Cliffs, New Jersey. For members with eccentric loading, see Article 10.36.

Singly symmetric and unsymmetric compression members, such as angles, or tees, and doubly symmetric compression members, such as cruciform or built-up members with very thin walls, may also require consideration of flexural-torsional and torsional buckling. Refer to the *Manual of Steel Constructio*n, Ninth Edition, 1989, American Institute of Steel Construction.

[d] See also Article 10.32.4.

[g] This shall apply to pins used primarily in axially loaded members, such as truss members and cable adjusting links. It shall not apply to pins used in members having rotation caused by expansion of deflection.

[i] When the area of holes deducted for high strength bolts or rivets is more than 15 percent of the gross area, that area in excess of 15 percent shall be deducted from the gross area in determining stress on the gross section. In determining gross section, any open holes larger than $1^1/_4$ inch diameter, such as perforations, shall be deducted.

TABLE 10.32.3A Allowable Stresses for Low-Carbon Steel Bolts and Power Driven Rivets (psi)

Type of Fastener	Tension[b]	Bearing[c]	Shear Bearing-Type Connection[b]
(A) Low-Carbon Steel Bolts[a] Turned Bolts (ASTM A 307) Ribbed Bolts	18,000	20,000	11,000
(B) Power-Driven Rivets (rivets driven by pneumatically or electrically operated hammers are considered power driven)			
Structural Steel Rivet Grade 1 (ASTM A 502 Grade 1)	—	40,000	13,500
Structural Steel Rivet (high strength) Grade 2 (ASTM A 502 Grade 2)	—	40,000	20,000

[a] ASTM A 307 bolts shall not be used in connections subject to fatigue.
[b] Applies to fastener cross sectional area based upon nominal body diameter.
[c] Applies to nominal diameter of fastener multiplied by the thickness of the metal.

TABLE 10.32.3B Allowable Stress for High-Strength Bolts or Connected Material (psi)

Load Condition	Allowable Stress
Applied Static Tension[a, b]	$0.315\ F_u^{\ d}$
Shear, F_v, on bolt with threads included in shear plane[c]	$0.16\ F_u^{\ d}$
Shear, F_v, on bolt with threads excluded from shear plane	$0.20\ F_u^{\ d}$
Bearing, F_p, on connected material in standard, oversize, short-slotted holes in any direction, or long-slotted holes parallel to the applied bearing force	$\frac{0.5L_cF_u}{d} \le F_u^{\ e,f,g}$
Bearing, F_p, on connected material in long-slotted holes perpendicular to the applied bearing force	$\frac{0.4L_cF_u}{d} \le 0.8F_u^{\ e,f,g}$

[a] Bolts must be tensioned to requirements of the Standard Specifications of California Department of Transportation
[b] See Article 10.32.3.4 for bolts subject to tensile fatigue
[c] In connection transmitting axial force whose length between extreme fasteners measured parallel to the line of force exceeds 50 inches, tabulated values shall be reduced 20 percent.
[d] F_u = specified minimum tensile strength of the fastener given in Table10.2C (psi)
[e] F_u = specified minimum tensile strength of connected mate rial (psi)
L_c = clear distance between the holes or between the hole and the edge of the material in the direction of the applied bearing force (in.)
d = nominal diameter of the bolt (in.)
[f] Connection using high-strength bolts in slotted holes with the load applied in a direction other than approximately normal (between 80 and 100 degrees) to the axis of the hole and connections with bolts in oversized holes shall be designed for resistance against slip in accordance with Article 10.32.3.2.1.
[g] Allowable bearing force for the connection is equal to the sum of the allowable bearing force for the individual bolts in the connection
[h] AASHTO M 164 (ASTM A 325) and AASHTO M253 (ASTM A 490) high-strength bolts are available in three types, designated as Types 1, 2,or 3. Type 3 shall be required on the plans when using unpainted AASHTO M 270 Grade 50W (ASTM A709 Grade 50W).

10.32.3.2.1 In addition to the allowable stress requirements of Article 10.32.3.2 the force on a slip-critical connection as defined in Article 10.24.1.4 shall not exceed the allowable slip resistance (P_s) of the connection according to:

$$P_s = K_h \mu T_b A_n N_b N_s \qquad (10\text{-}13)$$

where:

A_n = net cross section area of the bolt (in.^{2})
N_b = number of bolts in the joint
N_s = number of slip planes
T_b = required minimum bolt tension stress specified in the Standard Specifications of California Department of Transportation or equal to 70% of specified minimum tensile strength of bolts given in Table 10.2C (psi)
μ = slip coefficient specified in Table10.32.3C
K_h = hole size factor specified in Table 10.32.3D

Class A, B or C surface conditions of the bolted parts as defined in Table 10.32.3C shall be used in joints designated as slip-critical except as permitted in Article 10.32.3.2.2.

10.32.3.2.2 Subject to the approval of the Engineer, coatings providing a slip coefficient less than 0.33 may be used provided the mean slip coefficient is established by test in accordance with the requirements of Article 10.32.3.2.3, and the slip resistance per unit area is established.

10.32.3.2.3 Paint, used on the faying surfaces of connections specified to be slip-critical, shall be qualified by test in accordance with “Test Method to Determine the Slip Coefficient for Coatings Used in Bolted Joints” as adopted by the Research Council on Structural Connections. See Appendix A of *Allowable Stress Design Specification for Structural Joints Using ASTM A325 or A490 Bolts* published by the Research Council on Structural Connections.

TABLE 10.32.3C Slip Coefficient μ

Class Types	Contact Surface of Bolted Parts	μ
Class A	Clean mill scale and blast-cleaned surfaces with Class A coating	0.33
Class B	Blast-cleaned surfaces and blast-cleaned surfaces with Class B coating	0.5
Class C	Hot-dip galvanized surfaces roughened by hand wired brushing after galvanizing	0.33

Note: Coatings classified as Class A or Class B include those coatings which provide a mean slip coefficient not less than 0.33 or 0.5, respectively, as determined by Testing Method to Determine the Slip Coefficient for Coatings Used in the Bolted Joints. See Article 10.32.3.2.3.

TABLE 10.32.3D Hole Size Factor Slip K_h

Hole Types	K_h
Standard	1.0
Oversize and Short-slotted	0.85
Long-slotted holes with the slot perpendicular to the direction of the force	0.70
Long-slotted holes with the slot parallel to the direction of the force	0.60

10.32.3.3 Applied Tension, Combined Tension and Shear

10.32.3.3.1 High-strength bolts preferably shall be used for fasteners subject to tension or combined tension and shear.

10.32.3.3.2 Bolts required to support applied load by means of direct tension shall be so proportioned that their average tensile stress computed on the basis of nominal bolt area will not exceed the appropriate stress in Table 10.32.3B. The applied load shall be the sum of the external load and any tension resulting from prying action. The tension due to the prying action shall be

$$Q = \left[\frac{3b}{8a} - \frac{t^3}{20}\right] T \qquad (10\text{-}14)$$

where:

Q = the prying tension per bolt (taken as zero when negative) (lb.)
T = the direct tension per bolt due to external load (lb.)
a = distance from center of bolt under consideration to edge of plate (in.)
b = distance from center of bolt under consideration to toe of fillet of connected part (in.)
t = thickness of thinnest part connected (in.)

10.32.3.3.3 For combined shear and tension in slip-critical joints using high-strength bolts where applied forces reduce the total clamping force on the friction plane, the shear stress, f_v (psi), shall meet the following requirement:

$$f_v \le F_s\left(1 - 1.88 f_t / F_u\right) \qquad (10\text{-}15)$$

where:

f_t = calculated tensile stress in the bolt including any stress due to prying action (psi)
F_s = allowable slip stress (psi)
= $K_h m T_b$
F_u = specified minimum tensile strength of the bolt from Table 10.2C (psi)

10.32.3.3.4 Where rivets or high-strength bolts in bearing type connections are subject to both shear and tension, the tensile stress shall not exceed the reduced allowable tensile stress obtained from the following equations. The combined stresses shall meet the requirement of Equation (10-18).

for $f_v / F_v \le 0.33$

$$F_t' = F_t \qquad (10\text{-}16)$$

for $f_v / F_v > 0.33$

$$F_t' = F_t \sqrt{1 - \left(f_v / F_v\right)^2} \qquad (10\text{-}17)$$

$$f_v^2 + \left(k\, f_t\right)^2 \le F_v^2 \qquad (10\text{-}18)$$

where:

f_t = calculated tensile stress in rivet or bolt including any stress due to prying action (psi)
f_v = calculated shear stress in rivet or bolt (psi)
F_t = allowable tensile stress on rivet or bolt from Table 10.32.3A or Table 10.32.3B (psi)
F_t' = reduced allowable tensile stress on rivet or bolt due to the applied shear stress (psi)
F_v = allowable shear stress on rivet or bolt from Table 10.32.3A or Table 10.32.3B (psi)
k = a constant: 0.75 for rivets; 0.6 for high-strength bolts with threads excluded from shear plane

10.32.3.4 Fatigue

When subject to tensile fatigue loading, the tensile stress in the bolt due to the service load plus the prying force resulting from application of service load shall not exceed the following allowable stresses (psi). The nominal diameter of the bolt shall be used in calculating the bolt stress. The prying force shall not exceed 80 percent of the externally applied load.

Number of Cycles	AASHTO M 164 (ASTM A 325)	AASHTO M 235 (ASTM A 490)
Not more than 20,000	38,000	47,000
From 20,000 to 500,000	35,500	44,000
More than 500,000	27,500	34,000

10.32.4 Pins, Rollers, and Expansion Rockers

10.32.4.1 The effective bearing area of a pin shall be its diameter multiplied by the thickness of the material on which it bears. When parts in contact have different yield strength, F_y shall be the smaller value.

10.32.4.2 Design stresses for Steel Bars, Carbon Cold Finished Standard Quality, AASHTO M 169 (ASTM A 108), and Steel Forgings, Carbon and Alloy, for General Industrial Use, AASHTO M 102 (ASTM A 668), are given in Table 10.32.4.2A.

TABLE 10.32.4.2A Allowable Stresses—Steel Bars and Steel Forgings

AASHTO Designation with Size Limitations	—	M 102 To 20" in dia.	M 102 To 10" in dia.	M 102 To 20" in dia.
ASTM Designation Grade or Class	—	A 668 Class D	A 668 Class F	A 668[b] Class G
Minimum Yield Strength, psi	F_y	37,500	50,000	50,000
Stress in Extreme Fiber, psi	$0.80F_y$	30,000	40,000	40,000
Shear, psi	$0.40F_y$	15,000	20,000	20,000
Bearing on Pins not Subject to Rotation, psi[c]	$0.80F_y$	30,000	40,000	40,000
Bearing on Pins Subject to Rotation, psi (such as used in rockers and hinges)	$0.40F_y$	15,000	20,000	20,000

[b] May substitute rolled material of the same properties.
[c] This shall apply to pins used primarily in axially loaded members, such as truss members and cable adjusting links. It shall not apply to pins used in members having rotation caused by expansion or deflection.

10.32.5 Cast Steel, Ductile Iron Castings, Malleable Castings, and Cast Iron

10.32.5.1 Cast Steel and Ductile Iron

10.32.5.1.1 For cast steel conforming to specifications for Steel Castings for Highway Bridges, AASHTO M 192 (ASTM A 486), Mild-to-Medium-Strength Carbon-Steel Castings for General Application, AASHTO M103 (ASTM A27), and Corrosion-Resistant Iron-Chromium, Iron-Chromium-Nickel and Nickel-Based Alloy Castings for General Application, AASHTO M 163 (ASTM A 743), and for Ductile Iron Castings (ASTM A 536), the allowable stresses shall be in accordance with Table 10.32.5.1A.

10.32.5.1.2 When in contact with castings or steel of a different yield strength, the allowable bearing stress of the material with the lower yield strength shall govern. For riveted or bolted connections, Article 10.32.3 shall govern.

10.32.5.2 Malleable Castings

Malleable castings shall conform to specifications for Malleable Iron Castings, ASTM A 47 Grade 35018.

The following allowable stresses (psi) and modulus of elasticity (psi) shall be used:

Tension ………………………………. 18,000
Bending in Extreme Fiber ……………18,000
Modulus of Elasticity ……………25,000,000

10.32.5.3 Cast Iron

Cast iron castings shall conform to specifications for Gray Iron Castings, AASHTO M 105 (ASTM A 48), Class 30B. The following allowable stresses (psi) shall be used:

Bending in Extreme Fiber ……………..3,000
Shear …………………………………...3,000
Direct Compression, short columns ….12,000

10.32.5.4 *Deleted*

TABLE 10.32.5.1A Allowable Stresses—Cast Steel and Ductile Iron (psi)

AASHTO Designation	M 103	M 192	M 192		M 163	None
ASTM Designation	A 27	A 486	A 486		A 743	A 536
Class or Grade	70-36	70	90	120	CA-15	60-40-18
Minimum Yield Strength, F_y	36,000		60,000	95,000	65,000	40,000
Axial Tension	14,500		22,500	34,000	24,000	16,000
Tension in Extreme Fiber	14,500		22,500	34,000	24,000	16,000
Axial Compression, Short Columns	20,000		30,000	45,000	32,000	22,000
Compression in Extreme Fibers	20,000		30,000	45,000	32,000	22,000
Shear	09,000		13,500	21,000	14,000	10,000
Bearing, Steel Parts in Contact	30,000		45,000	68,000	48,000	33,000
Bearings on Pins not subject to Rotation	26,000		40,000	60,000	43,000	28,000
Bearings on Pins subject to Rotation (such as used in rockers and hinges)	13,000		20,000	30,000	21,500	14,000

10.32.6 Bearing on Masonry

10.32.6.1 The allowable bearing stress (psi) on the following types of masonry shall be:

Granite ……………………………………800
Sandstone and Limestone ……………400

10.32.6.2 The above bridge seat stress will apply only where the edge of the bridge seat projects at least 3 inches (average) beyond the edge of shoe or plate. Otherwise, the stresses permitted will be 75 percent of the above amounts.

10.32.6.3 For allowable bearing stress on concrete masonry, refer to Article 8.15.2.1.3.

10.33 ROLLED BEAMS

10.33.1 General

10.33.1.1 Rolled beams, including those with welded cover plates, shall be designed by the moment of inertia method. Rolled beams with riveted cover plates shall be designed on the same basis as riveted plate girders.

10.33.1.2 The compression flanges of rolled beams supporting timber floors shall not be considered to be laterally supported by the flooring unless the floor and fastenings are specially designed to provide adequate support.

10.33.2 Bearing Stiffeners

Suitable stiffeners shall be provided to stiffen the webs of rolled beams at bearings when the calculated shear stress in the web adjacent to the bearing exceeds 75 percent of the allowable shear stress for girder webs. See the related provisions of Article 10.34.6.

10.34 PLATE GIRDERS

10.34.1 General

10.34.1.1 Girders shall be proportioned by the moment of inertia method. For members primarily in bending, the entire gross section shall be used when calculating tensile and compressive stresses. Holes for high-strength bolts or rivets and/or open holes not exceeding $1^1/_4$ inches, may be neglected provided the area removed from each flange does not exceed 15 percent of that flange. That area in excess of 15 percent shall be deducted from the gross area.

10.34.1.2 The compression flanges of plate girders supporting timber floors shall not be considered to be laterally supported by the flooring unless the floor and fastenings are specially designed to provide support.

10.34.2 Flanges

10.34.2.1 Welded Girders

10.34.2.1.1 Each flange may comprise a series of plates joined end to end by full penetration butt welds. Changes in flange areas may be accomplished by varying the thickness and/or width of the flange plate, or by
+ adding cover plate. Varying the thickness and/or width of
+ the flange plate is preferred. Where plates of varying thicknesses or widths are connected, the splice shall be made in accordance with Article 10.18 and welds ground smooth before attaching to the web. The compression-flange width, *b*, on fabricated I-shaped girders preferably shall not be less than 0.2 times the web depth, but in no case shall it be less than 0.15 times the web depth. If the area of the compression flange is less than the area of the tension flange, the minimum flange width may be based on 2 times the depth of the web in compression rather than the web depth. The compression-flange thickness, *t*, preferably shall not be less than 1.5 times the web thickness. The width-to-thickness ratio, *b*/*t*, of flanges subject to tension shall not exceed 24.

10.34.2.1.2 When cover plates are used, they shall be designed in accordance with Article 10.13.

+ *10.34.2.1.3* The width-thickness ratio (*b*/*t*) of com-
+ pression flange plate shall not exceed the limiting values
+ specified in Table 10.34.2A.

+ *10.34.2.1.4* *Deleted*

TABLE 10.34.2A Limiting Width-Thickness Ratios for Compression Flanges of Plate Girders

Description of Component	Limiting (b/t)	When $f_b = 0.55\ F_y$	
		F_y (psi)	Limiting b/t
Compression flange plate of noncomposite welded plate girders	$\frac{3{,}250}{\sqrt{f_b}} \leq 24$ (10-19)	36,000 50,000 70,000 90,000 100,000	23 20 17 15 14
Compression flange plate of composite welded plate girders	$\frac{3{,}860}{\sqrt{f_{dl1}}} \leq 24$ (10-20)		
Outstanding legs of flange angles of noncomposite riveted or bolted girders	$\frac{1{,}625}{\sqrt{f_b}} \leq 12$ (10-21)	36,000 50,000 70,000 90,000 100,000	11.5 10 8.5 7.5 7.0
Outstanding legs of flange angles of composite riveted or bolted girders	$\frac{1{,}930}{\sqrt{f_{dl1}}} \leq 12$ (10-22)		

b = flange plate width for welded plate girders or outstanding leg width of flange angles for riveted and bolted girders (in.)
f_b = calculated compressive bending stress in flange (psi)
f_{dl1} = top flange compressive stress due to noncomposite dead load (psi)
F_y = specified minimum yield strength of the component under consideration (psi)
t = component plate thickness (in.)

10.34.2.1.5 In the case of a composite girder the width-thickness ratio (b/t) ratio of the top compression flange plate shall not exceed the limiting values specified in Table 10.34.2A.

10.34.2.2 Riveted or Bolted Girders

10.34.2.2.1 Flange angles shall form as large a part of the area of the flange as practicable. Side plates shall not be used except where flange angles exceeding $^7/_8$ inch in thickness otherwise would be required.

10.34.2.2.2 The width-thickness ratio (b'/t) of outstanding legs of flange angles in compression, except those reinforced by plates, shall not exceed the limiting values specified in Table 10.34.2A

10.34.2.2.3 Deleted

10.34.2.2.4 In the case of a composite girder the width-thickness ratio (b'/t) of outstanding legs of top flange angles in compression, except those reinforced by plates, shall not exceed the limiting values specified in Table 10.34.2A.

10.34.2.2.5 The gross area of the compression flange, except for composite design, shall be not less than the gross area of the tension flange.

10.34.2.2.6 Flange plates shall be of equal thickness, or shall decrease in thickness from the flange angles outward. No plate shall have a thickness greater than that of the flange angles.

10.34.2.2.7 At least one cover plate of the top flange shall extend the full length of the girder except when the flange is covered with concrete. Any cover plate that is not full length shall extend beyond the theoretical cutoff point far enough to develop the capacity of the plate or shall extend to a section where the stress in the remainder of the girder flange is equal to the allowable fatigue stress, whichever is greater. The theoretical cutoff point of the cover plate is the section at which the stress in the flange without that cover plate equals the allowable stress, exclusive of fatigue considerations.

10.34.2.2.8 The number of fasteners connecting the flange angles to the web plate shall be sufficient to develop the increment of flange stress transmitted to the flange angles, combined with any load that is applied directly to the flange.

10.34.2.2.9 Legs of angles 6 inches or greater in width, connected to web plates, shall have two lines of fasteners. Cover plates over 14 inches wide shall have four lines of fasteners.

10.34.3 Web Plates

10.34.3.1 Girders Not Stiffened Longitudinally

The girder without longitudinal stiffeners is usually preferred. The width-thickness ratio (D/t_w) of the web plate of plate girders without longitudinal stiffeners shall not exceed the limiting values specified in Table 10.34.3A.

10.34.3.1.1 Deleted

10.34.3.1.2 Deleted

10.34.3.2 Girders Stiffened Longitudinally

The width-thickness ratio (D/t_w) of the web plate of plate girders equipped with longitudinal stiffeners shall not exceed the limiting values specified in Table 10.34.3A.

10.34.3.2.1 Deleted

10.34.3.2.2 Deleted

TABLE 10.34.3A Limiting Width-Thickness Ratios for Web Plates of Plate Girders

Description of Web Plates	Limiting (D/t_w)	When $f_b = F_b$ or $f_v = F_v$: F_y (psi)	When $f_b = F_b$ or $f_v = F_v$: Limiting (D/t_w)
Without longitudinal stiffeners	$\frac{23{,}000}{\sqrt{f_b}} \le 170$ (10-23) (See Figure 10.34.3A)	36,000 50,000 70,000 90,000 100,000	165 140 115 105 100
With longitudinal stiffeners (Note: When $f_b = F_b$, limiting width-thickness ratio (D/t_w) shall apply to a symmetical girder stiffened with transverse stiffeners in combination with one longitudinal stiffener located a distance $D/5$ from the compression flange)	$\frac{4{,}050\sqrt{k}}{\sqrt{f_b}} \le 340$ (10-24) for $\frac{d_s}{D_c} \ge 0.4$ $\quad k = 5.17\left(\frac{D}{d_s}\right)^2 \ge 9\left(\frac{D}{D_c}\right)^2$ for $\frac{d_s}{D_c} < 0.4$ $\quad k = 11.64\left(\frac{D}{D_c - d_s}\right)^2$ for symmetrical girder see Figure 10.34.3.1A	36,000 50,000 70,000 90,000 100,000	327 278 235 207 196
Without transverse stiffeners	$\frac{8{,}510}{\sqrt{f_v}} \le 80$	36,000 50,000 70,000 90,000 100,000	78 66 56 50 47

D = depth of web or the clear unsupported distance between flange components (in.)
D_c = depth of web in compression calculated by summing the stresses from applicable stages of loadings (in.). In composite sections subjected to negative bending, D_c may be taken as the depth of the web in compression of the composite section without summing the stresses from various stage of loadings
d_s = distance from the centerline of a plate longitudinal stiffener or the gage line of an angle longitudinal stiffener to the inner surface or the leg of the compression flange component (in.)
f_b = calculated flange bending stress in the compression flange (psi)
f_v = calculated average shear stress in the gross section of the web plate (psi)
F_b = allowable bending stress (psi)
F_v = allowable shear stress (psi)
F_y = specified minimum yield strength of steel (psi)
k = buckling coefficient
t_w = web plate thickness (in.)

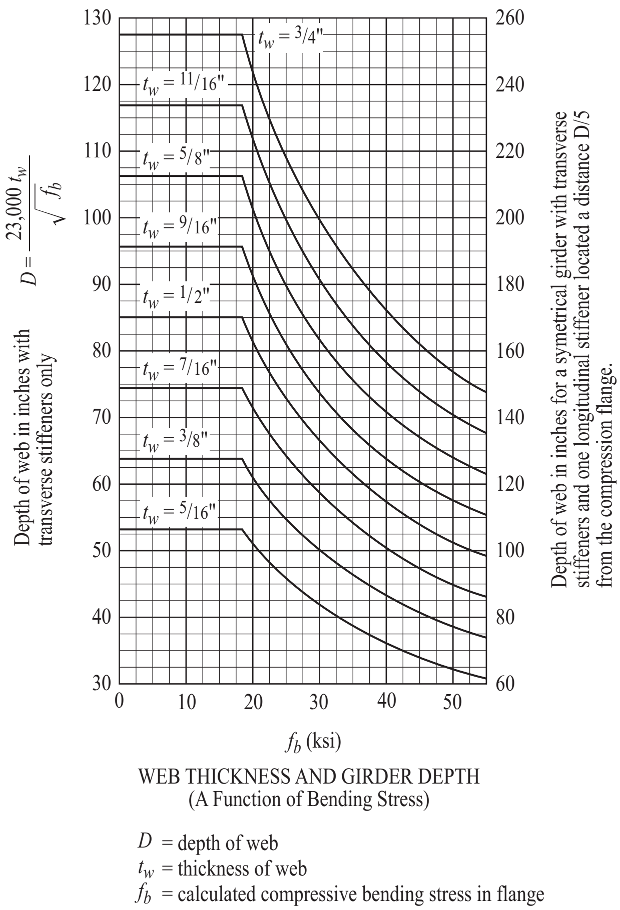

FIGURE 10.34.3.1A Web Thickness vs. Girder Depth for Non-Composite Symmetrical Sections

10.34.4 Transverse Intermediate Stiffeners

+ **10.34.4.1** Transverse intermediate stiffeners may be omitted if the average calculated shearing stress in the gross section of the web plate at the point considered, f_v, is less than the value given by the following equation:

$$F_v = \frac{7.33\times10^7}{\left(D/t_w\right)^2} \le \frac{F_y}{3} \qquad (10\text{-}25)$$

where:

+ F_v = allowable shear stress (psi)

10.34.4.2 Where transverse intermediate stiffeners are required, the spacing of the transverse intermediate stiffener shall be such that the calculated shearing stress will not exceed the value given by the following equation (the maximum spacing is limited to 3D subject to the handling requirements below): +

$$F_v = \frac{F_y}{3}\left[C + \frac{0.87\left(1-C\right)}{\sqrt{1+\left(d_o/D\right)^2}}\right] \qquad (10\text{-}26)$$

The constant C is equal to the buckling shear stress divided by the shear yield stress, and is determined as follows:

for $$\frac{D}{t_w} < \frac{6{,}000\sqrt{k}}{\sqrt{F_y}}$$

$$C = 1.0$$

for $$\frac{6{,}000\sqrt{k}}{\sqrt{F_y}} \le \frac{D}{t_w} \le \frac{7{,}500\sqrt{k}}{\sqrt{F_y}}$$

$$C = \frac{6{,}000\sqrt{k}}{\left(D/t_w\right)\sqrt{F_y}} \qquad (10\text{-}27)$$

for $$\frac{D}{t_w} > \frac{7{,}500\sqrt{k}}{\sqrt{F_y}}$$

$$C = \frac{4.5\times10^7\,k}{\left(D/t_w\right)^2\sqrt{F_y}} \qquad (10\text{-}28)$$

where:

$$k = 5 + \frac{5}{\left(d_o/D\right)^2}$$

d_o = spacing of intermediate stiffener (in.) +
F_y = specified minimum yield strength of the web plate (psi) +
+